Geotechnical Investigation

Proposed Residential Lot Split
Via Salvador, Valley Center
County of San Diego, California
(A.P.N. 188-321-22)

June 11, 2008

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GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL LOT SPLIT, VIA SALVADOR, VALLEY CENTER, COUNTY OF SAN DIEGO, CALIFORNIA (A.P.N. 188-321-22)

Pursuant to your request, Vinje and Middleton Engineering, Inc. has completed the attached Geotechnical Investigation Report for the project property.

The following report summarizes the results of our field investigation, including laboratory analyses and conclusions, and provides recommendations for the proposed residential lot split as understood. From a geotechnical engineering standpoint, it is our opinion that the property is suitable for the proposed lot split and future residential developments with the associated structures and improvements provided the recommendations presented in this report are incorporated into the design and construction of the project.

The conclusions and recommendations provided in this study are consistent with the site indicated geotechnical conditions and are intended to aid in preparation of final development plans and allow more accurate estimates of development costs.

If you have any questions or need clarification, please do not hesitate to contact this office. Reference to our **Job #08-287-P** will help to expedite our response to your inquiries.

We appreciate this opportunity to be of service to you.

VINJE & MIDDLETON ENGINEERING, INC.

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STATE OF CALIFORNIA

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PROPOSED RESIDENTIAL LOT SPLIT
VIA SALVADOR, VALLEY CENTER
COUNTY OF SAN DIEGO, CALIFORNIA
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I. INTRODUCTION

The property investigated for this work includes a nearly 5 acre parcel located within the community of Valley Center in San Diego County. A Regional Index Map showing the site location is enclosed with this report as Plate 1. We understand that the property is planned for a residential lot split into two parcels. Each parcel is proposed for the support of a single-family development with the associated structures and improvements. Consequently, the purpose of this investigation was to determine soil and goetechnical conditions at the future development areas and to ascertain their influence upon the proposed construction. Test pit digging, soil sampling, and laboratory analyses were among the activities conducted in conjunction with this effort which has resulted in the development recommendations presented herein.

II. SITE DESCRIPTION

The project property consists of a rectangular-shaped lot located at the eastern terminus of Via Salvador in Valley Center, east of Cole Grade Road. A Site Plan showing existing topographic conditions and the proposed two-lot subdivision is attached with this submittal as Plate 2.

The property is generally characterized by gently sloping terrain that is bisected by a north-south trending drainage flowline. The flowline appears to be a seasonal stream and was found to be in a dry condition at the time of our study. The site presently supports a modest to thick covering of native plants and bushes. Small man-made earthen mounds are present along dirt pathways. Gradients within the study locations are gentle and generally approach 10:1 (horizontal to vertical) maximum.

Local erosion resulting from uncontrolled runoff has resulted in the present drainage patterns and central flowline.

III. PROPOSED DEVELOPMENT

Preliminary development plans are depicted on the enclosed Plate 2. As shown, the property is planned for a two parcel split designated as Parcels 1 and 2 which will be developed for support of future single-family residential constructions. Sewage for both parcels will consist of septic disposal system.

Minor to modest grading efforts are proposed for the construction of level building pads and development of access driveway profiles. Vertical cut excavations and depth of filled ground will approach 5 feet maximum. Graded cut and fills slopes are programmed for 2:1 gradients maximum and will also approach maximum heights of 5 feet.

Detailed construction plans are not presently available. The use of conventional wood-frame with exterior stucco building construction supported on shallow foundations with stem walls and slab-on-grade floors, or slab-on-ground with turn-down footings is assumed herein for the purpose of this study.

IV. SITE INVESTIGATION

Geotechnical conditions at the study locations were chiefly determined by the excavation of 5 test pits dug with a tractor-mounted backhoe. All test excavations were logged by our project geologist who also retained representative soil and rock samples at selected locations and intervals for subsequent laboratory testing and engineering analyses. Test pit locations are shown on the Site Plan, Plate 2. Logs of the excavations are included as Plates 3-5. Laboratory results and engineering properties of the tested samples are summarized in following sections.

V. GEOTECHNICAL CONDITIONS

The proposed building pads and driveway alignments are sited upon natural terrain characterized by gently sloping ground underlain by crystalline bedrock units that are rooted in the southern California batholith. Landslides or areas of existing geologic instability are not in evidence at the property. The following geotechnical conditions are apparent:

A. Earth Materials

Bedrock: Crystalline bedrock units underlie the project areas at shallow depths. Exposures are typically fine to coarse grained gabbroic rocks that occur in a weathered and friable condition near the surface and grade blocky and hard at depth.

Project bedrock are competent units that will provide excellent support for new fills, structures, and improvements.

Topsoil: A thin cover of topsoil mantles the bedrock at the study site. Project topsoils are largely sandy deposits found typically in dry and loose conditions overall. Weathering of the bedrock has also resulted in a clay-bearing residual soil that is thought to be minor in overall quantities.

Details of site earth materials are given on the enclosed Test Pit Logs, Plates 3-5. The subsurface relationship of site deposits and proposed grades are also depicted on Geologic Cross-Sections enclosed with this report as Plate 6.

B. Groundwater and Surface Drainage

Surface water was not encountered to the depths explored and is not expected to impact grading activities or the future stability of the developed sites. As with all developed properties, the proper control of site surface drainage and irrigation waters is a critical component to overall stability of the graded building pads and embankments. Surface water should not pond upon graded surfaces, and irrigation water should not be excessive. Over-watering of site vegetation may also create perched water and the creation of excessively moist areas at finished lot surfaces.

C. Slope Stability

Significant slopes are not present at or in close proximity to the proposed building pads. Natural terrain at the study property exhibit no apparent instability. Landslides or other forms of geologic slope instability are not in evidence at the site.

Planned graded embankments are minor features that will range to five feet maximum in height. Graded embankments are programmed for 2:1 gradients and will be grossly stable to design heights provided our grading recommendations are flowed.

D. Rock Hardness

Bedrock underlying the project areas are generally weathered units which are expected to excavate to design grades and undercut depths with medium size dozers (Caterpillar D-6 or equal). Harder rocks and corestones were not encountered in our test excavations, but may be present resulting in the need for some local concentrated efforts and heavy ripping. Overall, however, unusual grading problems are not expected.

E. Faults/Seismicity

Faults or significant shear zones are not indicated on or near proximity to the project site.

As with most areas of California, the San Diego region lies within a seismically active zone; however, coastal areas of the county are characterized by low levels of seismic activity relative to inland areas to the east. During a 40-year period (1934-1974), 37 earthquakes were recorded in San Diego coastal areas by the California Institute of Technology. None of the recorded events exceeded a Richter magnitude of 3.7, nor did any of the earthquakes generate more than modest ground shaking or significant damages. Most of the recorded events occurred along various offshore faults which characteristically generate modest earthquakes.

Historically, the most significant earthquake events which affect local areas originate along well known, distant fault zones to the east and the Coronado Bank Fault to the west. Based upon available seismic data, compiled from California Earthquake Catalogs, the most significant historical event in the area of the study site occurred in 1800 at an estimated distance of 24.4 miles from the project area. This event, which is thought to have occurred along an off-shore fault, reached an estimated magnitude of 6.5 with estimated bedrock acceleration values of 0.60g at the project site. The following list represents the most significant faults which commonly impact the region. Estimated ground acceleration data compiled from Digitized California Faults (Computer Program EQFAULT VERSION 3.00 updated) typically associated with the fault is also tabulated.

TABLE 1

FAULT ZONE	DISTANCE FROM SITE	MAXIMUM PROBABLE ACCELERATION (R.H.)
Elsinore-Julian Fault	6.7 miles	0.134g
San Jacinto-Anza Fault	29.3 miles	0.075g
Rose Canyon Fault	26.6 miles	0.074g
Newport-Inglewood Fault	26.1 miles	0.070g

The location of significant faults and earthquake events relative to the study site are depicted on a Fault - Epicenter Map attached to this report as Plate 7.

More recently, the number of seismic events which affect the region appears to have heightened somewhat. Nearly 40 earthquakes of magnitude 3.5 or higher have been recorded in coastal regions between January 1984 and August 1986. Most of the earthquakes are thought to have been generated along offshore faults. For the most part, the recorded events remain moderate shocks which typically resulted in low levels of ground shaking to local areas. A notable exception to this pattern was recorded on July 13, 1986. An earthquake of magnitude 5.3 shook county coastal areas with moderate to locally heavy ground shaking resulting in $700,000 in damages, one death, and injuries to 30 people. The quake occurred along an offshore fault located nearly 30 miles southwest of Oceanside.

A series of notable events shook county areas with a (maximum) magnitude 7.4 shock in the early morning of June 28, 1992. These quakes originated along related segments of the San Andreas Fault approximately 90 miles to the north. Locally high levels of ground shaking over an extended period of time resulted; however, significant damages to local structures were not reported. The increase in earthquake frequency in the region remains a subject of speculation among geologists; however, based upon empirical information and the recorded seismic history of county areas, the 1986 and 1992 events are thought to represent the highest levels of ground shaking which can be expected at the study site as a result of seismic activity.

In recent years, the Rose Canyon Fault has received added attention from geologists. The fault is a significant structural feature in metropolitan San Diego which includes a series of parallel breaks trending southward from La Jolla Cove through San Diego Bay toward the Mexican border. Test trenching along the fault in Rose Canyon indicated that at that location the fault was last active 6,000 to 9,000 years ago. More recent work suggests that segments of the fault are younger having been last active 1000 - 2000 years ago. Consequently, the fault has been classified as active and included within an Alquist-Priolo Special Studies Zone established by the State of California.

Fault zones tabulated in the preceding table are considered most likely to impact the region of the study site during the lifetime of the project. The faults are periodically active and capable of generating moderate to locally high levels of ground shaking at the site. Ground separation as a result of seismic activity is not expected at the property.

A site specific probabilistic estimation of peak ground acceleration was also performed using the FRISKSP (T. Blake, 2000) computer program. Based upon Boore it. al (1997) attenuation relationship, a 10 percent probability of exceedance in 50 years was estimated to produce a site specific peak ground acceleration of 0.27g (Design-Basis Earthquake, DBE). The results were obtained from the corresponding probability of exceedance versus acceleration curve.

F. Seismic Ground Motion Values

For design purposes, site specific seismic ground motion values were determined as part of this investigation in accordance with the California Building Code (CBC). The following parameters are consistent with the indicated project seismic environment and our experience with similar earth deposits in the vicinity of the project site, and may be utilized for project design work:

TABLE 2

Site Class	S_S	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
B	1.5	0.584	1	1	1.5	0.584	1	0.389
According to Chapter 16, Section 1613 of the 2007 California Building Code.								

Explanation:

S_S: Mapped MCE, 5% damped, spectral response acceleration parameter at short periods.
S_1: Mapped MCE, 5% damped, spectral response acceleration parameter at a period of 1-second.
F_a: Site coefficient for mapped spectral response acceleration at short periods.
F_v: Site coefficient for mapped spectral response acceleration at 1-second period.
S_{MS}: The MCE, 5% damped, spectral response acceleration at short periods adjusted for site class effects ($S_{MS}=F_aS_S$).
S_{M1}: The MCE, 5% damped, spectral response acceleration at a period of 1-second adjusted for site class effects ($S_{M1}=F_vS_1$).
S_{DS}: Design, 5% damped, spectral response acceleration parameter at short periods ($S_{DS}=\frac{2}{3}S_{MS}$).
S_{D1}: Design, 5% damped, spectral response acceleration parameter at a period of 1-second ($S_{D1}=\frac{2}{3}S_{M1}$).

G. Geologic Hazards

Geologic hazards are not presently indicated at the project site. The most significant geologic hazards at the property will be those associated with ground shaking in the event of a major seismic event. Liquefaction or related ground rupture failures are not anticipated.

H. Field and Laboratory Tests and Test Results

Earth deposits encountered in our exploratory test excavations were closely examined and sampled for laboratory testing. Based upon our test pits and field exposures site soils have been grouped into the following soil types:

TABLE 3

Soil Type	Description
1	Brown silty fine to medium sand (topsoil)
2	Red brown to grey fine to coarse sand (bedrock)
3	Olive brown fine to medium sand with a trace of clay (topsoil)

The following tests were conducted in support of this investigation:

1. **Maximum Dry Density and Optimum Moisture Content:** The maximum dry density and optimum moisture content of Soil Types 1 and 2 were determined in accordance with ASTM D-1557. The results are presented in Table 4.

TABLE 4

Location	Soil Type	Maximum Dry Density (γ_m-pcf)	Optimum Moisture Content (ω_{opt}-%)
TP-3 @ 4'	2	129.8	12.3
TP-4 @ 1'	1	133.3	9.8

2. **Moisture-Density Tests (Undisturbed Chunk Samples):** In-place dry density and moisture content of representative soil deposits beneath the site were determined from relatively undisturbed chunk samples using the water displacement test method. Results are presented in Table 5 and tabulated on the attached Test Trench Logs.

TABLE 5

Sample Location	Soil Type	Field Moisture Content (ω-%)	Field Dry Density (γ_d-pcf)	Max. Dry Density (γ_m-pcf)	In-Place Relative Compaction	Degree of Saturation S (%)
TP-1 @ 2'	1	6	112.1	133.3	84	32
TP-2 @ 2'	2	5	136.5	129.8	100+	59
TP-4 @ 2'	1	7	111.1	133.3	83	37
TP-4 @ 3½'	3	16	106.4	-	-	74
TP-5 @ 2'	2	3	90.4	133.3	68	9
Assumptions And relationships: In-place Relative Compaction = ($\gamma_d \div \gamma_m$) X100 Gs = 2.70 e = (Gs $\gamma_\omega \div \gamma_d$) - 1 S = ($\omega$ Gs) $\div$ e						

3. **Expansion Index Test:** One expansion index (EI) test was performed on a representative sample of Soil Type 1 in accordance with the ASTM D-4829. The test result is presented in Table 6.

TABLE 6

Sample Location	Soil Type	Molded ω (%)	Degree of Saturation (%)	Final ω (%)	Initial Dry Density (PCF)	Measured EI	EI 50% Saturation
TP-4 @ 1'	1	7.6	44.9	15.3	115.6	0	0

(ω) = moisture content in percent.

$EI_{50} = EI_{meas} - (50 - S_{meas}) ((65 + EI_{meas}) \div (220 - S_{meas}))$

Expansion Index (EI)	Expansion Potential
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
〉130	Very High

4. **Direct Shear Test:** One direct shear test was performed on a representative sample of Soil Type 1. The prepared specimen was soaked overnight, loaded with normal loads of 1, 2, and 4 kips per square foot respectively, and sheared to failure in an undrained condition. The test result is presented in Table 7.

TABLE 7

Sample Location	Soil Type	Sample Condition	Wet Density (γ_w-pcf)	Angle of Int. Fric. (Φ-Deg.)	Apparent Cohesion (c-psf)
TP-4 @ 1'	1	remolded to 90% of γ_m @ % ω_{opt}	132.5	34	0

5. **pH and Resistivity Test:** pH and resistivity of a representative sample of Soil Type 1 was determined using "Method for Estimating the Service Life of Steel Culverts," in accordance with the California Test Method (CTM) 643. The test result is tabulated in Table 8.

TABLE 8

Sample Location	Soil Type	Minimum Resistivity (OHM-CM)	pH
TP-4 @ 1'	1	8400	6.4

6. **Sulfate Test:** A sulfate test was performed on a representative sample of Soil Type 1 in accordance with the California Test Method (CTM) 417. The test result is presented in Table 9.

TABLE 9

Sample Location	Soil Type	Amount of Water Soluble Sulfate In Soil (% by Weight)
TP-4 @ 1'	1	<0.001

7. **Chloride Test:** A chloride test was performed on a representative sample of Soil Type 1 in accordance with the California Test Method (CTM) 422. The test result is presented in Table 10.

TABLE 10

Sample Location	Soil Type	Amount of Water Soluble Chloride In Soil (% by Weight)
TP-4 @ 1'	1	<0.001

8. **R-Value Test:** One R-value test was performed on a representative sample of Soil Type 1 in accordance with the California Test Method (CTM) 301. The test result is presented in Table 11.

TABLE 11

Location	Soil Type	Description	R-Value
TP-1 @ 1'	1	brown silty fine to medium sand	48

VI. SITE CORROSION ASSESSMENT

A site is considered to be corrosive to foundation elements, walls and drainage structures if one or more of the following conditions exists:

* Sulfate concentration is greater than or equal to 2000 ppm (0.2% by weight).
* Chloride concentration is greater than or equal to 500 ppm (0.05 % by weight).
* pH is less than 5.5.

For structural elements, the minimum resistivity of soil (or water) indicates the relative quantity of soluble salts present in the soil (or water). In general, a minimum resistivity value for soil (or water) less than 1000 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion. Appropriate corrosion mitigation measures for corrosive conditions should be selected depending on the service environment, amount of aggressive ion salts (chloride or sulfate), pH levels and the desired service life of the structure.

Laboratory test results performed on selected representative site samples indicate that the minimum resistivity is more than 1000 ohm-cm suggesting presence of low quantities of soluble salts. Test results further indicated pH level is less than 5.5, sulfate concentration is less than 2000 ppm, and chloride concentration level is less than 500 ppm. Based on the results of the corrosion analyses, the project site is considered non-corrosive. The project site is not located within 1000 feet of salt or brackish water.

Based upon the result of the tested soil sample, the amount of water soluble sulfate (SO_4) was found to be less than 0.001 percent by weight which is considered negligible according to ACI 318, Table 4.3.1. Portland cement Type II may be used. Table 12 is appropriate based on the pH-Resistivity test result:

TABLE 12

Design Soil Type	Gage	18	16	14	12	10	8
1	Years to Perforation of Steel Culverts	23	29	36	50	64	78

VII. CONCLUSIONS

Based upon the foregoing investigation, the planned lot split for two residential parcels substantially as proposed is feasible from a geotechnical viewpoint. The property is underlain by dense, stable bedrock units at shallow depths.

Geotechnical factors presented below are unique to the project site and will most influence grading procedures:

* The study areas are underlain by competent crystalline bedrock units which will adequately support new fills, embankments, structures and improvements. Geologic instability is not indicated at the site.

* Gabbroic bedrock underlying the property are weathered units and excavation difficulties are not expected. Based upon our site observations and available subsurface exposures, final design grades and undercut depths are expected to be achieved using medium bulldozers (Caterpillar D-6 or equivalent). Locally, harder bedrock units or corestones may also be encountered requiring added and more concentrated efforts.

* Weathered bedrock excavations will predominantly generate good quality sandy to gravelly fills. Corestones and larger rock, if encountered during grading, should be excluded from project fills and properly disposed of as specified below. The upper soil mantle are also sandy deposits which may locally include some clay-bearing soils. Minor clayey soils, if locally encountered from the site removals, should be thoroughly mixed with an abundance of sandy soils available from the project excavations as directed in the field.

* Existing upper soil cover at the site consists of loose to very loose and dry deposits which are not suitable in their present condition for structural support. Regrading of these deposits is recommended in the following sections in order to construct safe and stable building surfaces. Added removals of cut ground and its reconstruction to design grades with compacted fills will also be necessary in the case of cut-fill transition pads so that uniform bearing soils conditions are constructed throughout the buildings and improvement surfaces.

* Based on subsurface exploratory excavations and remedial grading recommendations given herein, final bearing soils are expected to chiefly consist of gravelly silty sand (SM/SW) deposits with very low expansion potential (expansion index less than 21) based on ASTM D-4829 classification. Actual classification and expansion characteristic of the finished grade soil mix can only be provided in the final as-graded compaction report based on proper testing of foundation bearing and slab subgrade soils.

* Planned graded slopes are minor embankments less than 5 feet high maximum and natural terrain at the project site geologically stable, gently sloping ground. Slope stability is not considered a major geotechnical concern at the study property. Minor graded slopes should be constructed as specified in the following sections.

* Natural groundwater is not expected to be a factor in the site development. However, the proper control of surface drainage and storm water including the seasonal flow within the nearby flow line is a critical component to the overall site, embankments and building performance. Surface water should not pond upon graded surfaces, and irrigation water should not be excessive. Over-watering of site vegetation may also create perched water and the creation of excessively moist areas at finished surfaces and should be avoided.

* Site grading and earthwork constructions will not impact the adjacent properties provided our recommendations are incorporated into the final designs and implemented during the construction phase. Added field recommendations, however, may also be necessary and should be given by the project geotechnical consultant for the protection of adjacent properties and should be anticipated.

* Post construction settlement of site fill deposits after completion of grading works as specified herein, is not expected to exceed approximately ½-inch and should occur below the heaviest loaded footing(s). The magnitude of post construction differential settlements of site fill deposits as expressed in terms of angular distortion is not anticipated to exceed ¼-inch between similar adjacent structural elements.

* Soil collapse, liquefaction and seismically induced settlements will not be a factor in the development of the project property provided our remedial grading recommendations are implemented at the site.

VIII. RECOMMENDATIONS

The following recommendations are consistent with the indicated geotechnical conditions at the project site and should be reflected in final plans and implemented during the construction phase. Added or modified recommendations may also be appropriate and can be provided at the final plan review phase:

A. Grading and Earthworks

Cut-fill and remedial grading techniques may be used in order to achieve final design grades and construct stable building surfaces for the support of the planned structures and improvements. All grading and earthworks should be completed in accordance with the Appendix "J" of the California Building Code (CBC), County of San Diego Grading Ordinances, the Standard Specifications for Public Works Construction and the requirements of the following sections:

1. **Existing Underground Utilities and Structures:** All existing underground waterlines, leach field lines, storm drains, utilities, tanks, structures and improvements at or nearby the project construction sites should be thoroughly potholed, identified and marked prior to the initiation of the actual grading work. Specific geotechnical engineering recommendations may be required based on the actual field locations and invert elevations, backfill conditions and proposed grades in the event of a grading or construction conflict.

   Utility lines may need to be temporarily redirected, if necessary, prior to earthwork operations and reinstalled upon completion of grading operations. Alternatively, permanent relocations may be appropriate as shown on the approved plans.

   Abandoned lines, irrigation pipes and conduits should be properly removed, capped or sealed off to prevent any potential for future water infiltrations into the foundation bearing, subgrade and backfill soils. Voids created by the removals of the abandoned underground pipes, tanks and structures should be properly backfilled with compacted fills in accordance with the requirements of this report.

2. **Clearing and Grubbing:** Remove surface vegetation, trees, roots, stumps, boulder rocks, construction debris, and all other unsuitable materials and deleterious matter from all areas of proposed new fills, improvements and structures plus 10 feet outside the perimeter, where possible and as approved in the field.

Debris generated from the removals and demolition of the site existing structures, improvements, pavings, and abandoned underground facilities should also be properly removed and disposed of from the site. Trash, vegetation and debris should not be allowed to occur or contaminate new site fills and backfills.

The prepared grounds should be inspected and approved by the project geotechnical engineer or his designated field representative prior to remedial grading and earthworks.

3. **Removals and Remedial Grading:** Site upper topsoil mantle in the areas of planned new fills, embankments, structures and improvements plus 10 feet outside the perimeter, where possible and as directed in the field, should be removed to the underlying competent bedrock and placed back as properly compacted fills. Recommended remedial grading should also include removals and recompaction of all existing site fills and earthen mounds, where they are encountered or occur, as directed in the field.

   Actual removal depths should be established by the project geotechnical consultant based on field observations of subsurface exposures developed during the remedial grading operations. Approximate removal depths in the vicinity of individual exploratory test sites are shown in Table 13. The tabulated values are typical and subject to field changes based on actual exposures. Locally deeper removals may be necessary and should be anticipated.

TABLE 13

Test Pit Location	Total Depth of Boring (ft)	Estimated Depth of Over-Excavation (ft)	Estimated Depth of Groundwater (ft)	Comments
TP-1	3½'	2½'	not encountered	Driveway areas on Parcel 1. Depth of undercut may govern.
TP-2	3½'	1'	not encountered	Parcel 1 pad fill areas. Depth of fill slope toe keyway or benching may govern.
TP-3	4½'	1½'	not encountered	Parcel 1 pad cut areas. Depth of undercut may govern.
TP-4	5'	4'	not encountered	Parcel 2 pad cut areas. Depth of undercut may govern.
TP-5	4'	3'	not encountered	Parcel 2 pad fill areas. Depth of fill slope toe keyway or benching may govern.

Notes:
1. All depths are measured from the existing ground levels.
2. Remove and recompact all existing site fills and earthen mounds as directed in the field.
3. Actual depths may vary at the time of construction based on field conditions.
4. Bottom of all removals should be prepared, ripped and recompacted in-place as directed in the field.
5. All grounds steeper than 5:1 receiving fills/backfills should be properly benched and keyed as directed in the field.
6. Exploratory trenches excavated in connection with our study at the indicated locations were backfilled with loose and uncompacted deposits. The loose/uncompacted backfill soils within these trenches shall also be re-excavated and placed back as properly compacted fills as a part of the project grading operations.

4. **Excavations Characteristics:** Very hard bedrock units requiring specialized excavation techniques were not encountered in our exploration trenches. Underlying rocks are weathered units which are expected to be excavated to design grades and undercut depths with light to moderate efforts using medium bulldozers (Caterpillar D-6 or equivalent). Harder, less weathered units or corestones may also be locally encountered requiring added and more concentrated excavation efforts. Harder bedrock units and corestones, if encountered, typically generate larger rock sizes which should be selectively excluded from the site fills and backfills.

5. **Cut - Fill Transitions and Undercuts:** Ground transition from excavated cut to compacted fills should not be permitted underneath the proposed structures and improvements. Building and structural foundations as well as on-grade improvements should be supported entirely on compacted fills or uniformly founded on undisturbed competent bedrock units. Transition pads will require special treatment. The cut portion of the cut-fill pads plus 10 feet outside the perimeter, where possible and as directed in the field, should be undercut to a sufficient depth to provide for a minimum 3 feet of compacted fill mat below rough finish grades, or at least 12 inches of compacted fill beneath the deepest footing(s) whichever is more. In the roadways, driveway, parking and on-grade slabs/improvement transition areas there should be a minimum 12 inches of compacted soils below rough finish subgrade.

   Undercutting the cut portion of the building pad will also accommodate excavation of foundation trenches and underground utilities. In the case of deeper utility or storm drain trenches, undercutting to a minimum 8 inches below the proposed inverts should be considered.

6. **Fill Materials and Compaction:** Soils generated from excavations of site topsoil cover and weathered bedrock units will predominantly consist of gravelly to sandy materials which are suitable fore reuse as new site fills and backfills. Some clay-bearing soils may be encountered during topsoil removals which are expected to be minor in overall quantities. Minor clayey soils, where they occur,

should be thoroughly mixed with an abundance of onsite sandy granular soils to create a very low expansive mixture as directed and approved in the field. Larger rock sizes (more than 6-inches in maximum dimension), if encountered or generated from the harder bedrock excavations, should also be selectively excluded from the site fills and backfills. Locally clayey to gravelly fills typically require added processing and moisture conditioning efforts for manufacturing a suitable fill mixture.

Project fills and backfills should be clean deposits free of vegetation, roots, trash, debris, organic materials, deleterious matter, larger than 6 inches rock sizes (plus 3 inches for trench and wall backfills) and include at least 40% finer than #4 sieve materials by weight as approved in the field. Uniform bearing and subgrade soil conditions should be constructed at the site by the grading operations. All site fills, and wall and trench backfills should be adequately processed, moisture conditioned to slightly (2%) above optimum levels as directed in the field, thoroughly mixed, manufactured into a uniform mixture, placed in thin (8 inches maximum) uniform horizontal lifts and mechanically compacted to a minimum 90% of the corresponding laboratory maximum dry density (ASTM D-1557), unless otherwise specified.

7. **Shrinkage, Bulking and Import Soils:** Site upper loose topsoils cover may be expected to shrink approximately 5% to 15%, while soils generated from the site weathered bedrock excavations bulk nearly 5% to 10% on a volume basis when compacted as specified herein.

   Import soils, if required to complete grading and backfilling, should be sandy granular non-corrosive deposits (SM/SW) with very low expansion potential (100% passing 1-inch sieve, more than 50% passing #4 sieve and less than 18% passing #200 sieve with expansion index less than 21). Import soils should be inspected, tested as necessary, and approved by the project geotechnical engineer prior to delivery to the site. Import soils should also meet or exceed engineering characteristic and soil design parameters as specified in the following sections.

8. **Permanent Graded Slopes:** Major graded slopes are not planned at the site and the proposed manufactured slopes are expected to be minor on the order of 5 feet maximum. Graded slopes should be programmed for 2:1 gradients maximum. Graded slopes constructed as recommended herein will be grossly stable with respect to deep seated and surficial failures for the indicated design maximum heights and gradients.

All fill slopes should be provided with a lower toe keyway. The keyway should maintain a minimum depth of 2 feet into the competent bedrock with a minimum width of 12 feet unless as approved in the field. The keyway should expose competent and stable formational units throughout with the bottom heeled back a minimum of 2% into the natural hillside and observed and approved by the project geotechnical consultant. Additional level benches should be constructed into the natural hillside as the slope construction progresses. Fill and stability slopes should be compacted to minimum 90% of the laboratory standard out to the slope face as specified. Over-building and cutting back to the compacted core, or backrolling at a maximum 4-foot vertical increments and "track-walking" at the completion of grading is recommended for site fill slope construction. Geotechnical engineering observations and testing will be necessary to confirm adequate compaction levels within the fill slope face.

Cut slopes are expected to expose topsoils and competent bedrock units at finish surfaces. Track-walking of the cut embankment will be required in the areas where loose topsoils are exposed to achieve 90% compaction levels within the slope face as directed in the field. All cut slopes should be observed and approved by the project geotechnical consultant during the grading. More specific recommendations may be necessary and should be provided at that time in the event adverse geologic conditions or unfavorable features are noted.

9. **Surface Drainage and Erosion Control:** A critical element to the continued stability of the building pads and slopes is an adequate surface drainage system and protection of the slope face. Surface, storm water and seasonal flow within the nearby stream shall not be allowed to impact the graded building pads and developed improvement sites. This can most effectively be achieved by appropriate vegetation cover and the installation of the following systems:

   * Concentrated surface run-off or flow should not be allowed to occur on or near the project graded sites. Drainage and storm water control structures and improvement should be installed per approved plans. Final pad grades should also maintain adequate elevations above the highest anticipated storm water levels caused by the design storm event as determined appropriate by the project design consultant.

   * Building pad surface run-off should be collected and directed away from the planned buildings and improvements to a selected location in a controlled manner. Area drains should be installed.

   * The finished slope should be planted soon after completion of grading. Unprotected slope faces will be subject to severe erosion and should not be allowed. Over-watering of the slope faces should also not be allowed. Only the amount of water to sustain vegetation should be provided.

* Temporary erosion control facilities and silt fences should be installed during the construction phase periods and until landscaping is fully established as indicated and specified on the approved project grading/erosion plans.

10. **Engineering Observations:** All grading and earthworks operations including removals, suitability of earth deposits used as compacted fills and backfills, and compaction procedures should be continuously observed and tested by the project geotechnical consultant and presented in the final as-graded compaction report. The nature of finished bearing and subgrade soils should be confirmed in the final compaction report at the completion of grading.

    Geotechnical engineering observations should include but not limited to the following:

    * Initial observation - After the clearing limits have been staked but before grading/brushing starts.

    * Bottom of toe keyway/over-excavation observation - After competent bedrock is exposed and prepared to receive fill or backfill but before fill or backfill is placed.

    * Cut/excavation observation - After the excavation is started but before the vertical depth of excavation is more than 5 feet. Local and Cal-OSHA safety requirements for open excavations apply.

    * Fill/backfill observation - After the fill/backfill placement is started but before the vertical height of fill/backfill exceeds 2 feet. A minimum of one test shall be required for each 100 lineal feet maximum in every 2 feet vertical gain, with the exception of wall backfills where a minimum of one test shall be required for each 30 lineal feet maximum. Wall backfills should consist of minus 3-inch materials and also mechanically compacted to a minimum of 90% compaction levels unless otherwise specified or directed in the field. Finish rough and final pad grade tests shall be required regardless of fill thickness.

    * Foundation trench observation - After the foundation trench excavations but before steel placement.

    * Foundation bearing/slab subgrade soils observation - Prior to the placement of concrete for proper moisture and specified compaction levels.

    * Geotechnical foundation/slab steel observation - After the steel placement is completed but before the scheduled concrete pour.

* Underground utility/plumbing trench observation - After the trench excavations but before placement of pipe bedding or installation of the underground facilities. Local and Cal-OSHA safety requirements for open excavations apply. Observation of pipe bedding may also be required by the project geotechnical engineer.

* Underground utility/plumbing trench backfill observation - After the backfill placement is started above the pipe zone but before the vertical height of backfill exceeds 2 feet. Testing of the backfill within the pipe zone may also be required by the governing agencies. Pipe bedding and backfill materials shall conform to the governing agencies' requirements and project soils report if applicable. All trench backfills shall consist of minus 3-inch particles and mechanically compacted to a minimum of 90% compaction levels unless otherwise specified. Plumbing trenches more than 12 inches deep maximum under the floor slabs should also be mechanically compacted and tested for a minimum of 90% compaction levels. Flooding or jetting techniques as a means of compaction method should not be allowed.

* Pavement/improvements base and subgrade observation - Prior to the placement of concrete or asphalt for proper moisture and specified compaction levels.

B. Foundations and Slab-on-Grades

The following recommendations are consistent with very low expansive (expansion index less than 21) gravelly silty sand (SM/SW) foundation bearing soils and site specific geotechnical conditions. Additional recommendations may also be required and should be given at the plan review phase. All design recommendations should be further confirmed and/or revised at the completion of rough grading based on the expansion characteristics of the foundation bearing soils and as-graded site geotechnical conditions, and presented in the final as-graded compaction report:

1. Proposed new buildings may be supported on shallow stiff foundations with stem walls and slab-on-grade floors or slab-on-ground with turned-down footings. The shallow foundations should be uniformly supported on certified compacted fills.

2. Continuous strip stem wall foundations and turned-down footings should be sized at least 15 inches wide and 18 inches deep for single and two-story structures. Spread pad footings should be at least 24 inches square and 12 inches deep. Specified depths are measured from the lowest adjacent ground surface, not including the sand/gravel layer beneath floor slabs. Exterior continuous foundations or turned-down footings should enclose the entire building perimeter.

Continuous interior and exterior stem wall foundations should be reinforced with minimum four #4 reinforcing bars. Place 2-#4 bars 3 inches above the bottom of the footings and 2-#4 bars 3 inches below the top of the stem wall. Turned-down footings should be reinforced with minimum 2-#4 bars at the top and 2-#4 bars at the bottom. Reinforcement details for spread pad footings should be provided by the project architect/structural engineer.

3. All interior slabs should be a minimum of 4 inches in thickness, reinforced with #3 reinforcing bars spaced 18 inches on center each way, placed mid-height in the slab. Slabs should be underlain by 4 inches of clean sand (SE 30 or greater) which is provided with a well performing moisture barrier/vapor retardant (15-mil plastic) placed mid-height in the sand.

   Provide "softcut" contraction/control joints consisting of sawcuts spaced 10 feet on centers each way for all interior slabs. Cut as soon as the slab will support the weight of the saw and operate without disturbing the final finish which is normally within 2 hours after final finish at each control joint location or 150 psi to 800 psi. The sawcuts should be a minimum of 1-inch in depth but should not exceed 1¼-inches deep maximum. Anti-ravel skid plates should be used and replaced with each blade to avoid spalling and raveling. Avoid wheeled equipments across cuts for at least 24 hours.

   Provide re-entrant corner reinforcement for all interior slabs. Re-entrant corners will depend on slab geometry and/or interior column locations. The enclosed Plate 8 may be used as a general guideline.

4. Adequate setback or deepened foundations shall be required for all foundations constructed on or near the top of descending slopes to maintain minimum horizontal distances to daylight or adjacent slope face. There should be a minimum of 7 feet horizontal setback from the bottom outside edge of the footing to daylight for foundations unless otherwise specified or approved. A minimum of 10 feet horizontal distances or set back shall be required for sensitive structures and improvements which cannot tolerate minor movements (including swimming pools and spas or portions thereof) located near the top of project descending slopes.

5. Foundation trenches and slab subgrade soils should be observed and tested for proper moisture and specified compaction levels and approved by the project geotechnical consultant prior to the placement of steel reinforcement or concrete pour.

C. Exterior Concrete Slabs / Flatworks

1. All exterior slabs (walkways, and patios) should be a minimum 4 inches in thickness reinforced with 6x6/10x10 welded wire mesh carefully placed mid-height in the slab. Subgrade soils beneath all exterior slabs should maintain a minimum 90% compaction levels as tested and approved in the field.

2. In construction practices where the reinforcements are cut at the construction joints, slab panels should be tied together with minimum 18 inches long #3 dowels (dowel baskets) at 18 inches on centers placed mid-height in the slab (9 inches on either side of the joint).

3. Provide "tool joint" or "softcut" contraction/control joints spaced 10 feet on center (not to exceed 12 feet maximum) each way. The larger dimension of any panel shall not exceed 125% of the smaller dimension. Tool or cut as soon as the slab will support weight and can be operated without disturbing the final finish which is normally within 2 hours after final finish at each control joint location or 150 psi to 800 psi. Tool or softcuts should be a minimum of 1-inch but should not exceed 1¼-inches deep maximum. In case of softcut joints, anti-ravel skid plates should be used and replaced with each blade to avoid spalling and raveling. Avoid wheeled equipments across cuts for at least 24 hours.

4. All exterior slab designs should be confirmed in the final as-graded compaction report.

5. Subgrade soils should be tested for proper moisture and specified compaction levels and approved by the project geotechnical consultant prior to the placement of concrete.

D. Soil Design Parameters

The following soil design parameters are based upon tested representative samples of onsite earth deposits. All parameters should be re-evaluated when the characteristics of the final as-graded soils have been specifically determined:

* Design soil unit weight = 133 pcf.
* Design angle of internal friction of soil = 34 degrees.
* Design active soil pressure for retaining structures =38 pcf (EFP), level backfill, cantilever, unrestrained walls.
* Design at-rest soil pressure for retaining structures = 58 pcf (EFP), non-yielding, restrained walls.
* Design passive soil resistance for retaining structures = 450 pcf (EFP), level ground surface on the toe side.

* Design coefficient of friction for concrete on soils = 0.42.
* Net allowable foundation pressure (minimum 15 inches wide by 18 inches deep footings) = 2000 psf.
* Allowable lateral bearing pressure (all structures except retaining walls) = 200 psf/ft .

Notes:

* Use a minimum safety factor of 1.5 for wall over-turning and sliding stability. However, because large movements must take place before maximum passive resistance can be developed, a safety factor of 2 may be considered for sliding stability where sensitive structures and improvements are planned near or on top of retaining walls.

* When combining passive pressure and frictional resistance the passive component should be reduced by one-third.

* The net allowable foundation pressures provided herein were determined based on the specified foundation depths and widths. The indicated values may be increased by 20% for each additional foot of depth and 20% for each additional foot of width to a maximum of 5500 psf, if needed. The allowable foundation pressures provided herein also applies to dead plus live loads and may be increased by one-third for wind and seismic loading.

* The allowable lateral bearing earth pressures may be increased by the amount of the designated value for each additional foot of depth to a maximum of 1500 pounds per square foot.

E. Asphalt and PCC Pavement Design

1. **Asphalt Paving:** The following asphalt pavement structural sections are based on a tested subgrade R-value of 48 and indicated traffic Indices (TI), and may be considered for onsite asphalt pavings outside public and private right-of-way. A minimum section of 3 inches asphalt (AC) over 4 inches of Class 2 crushed aggregate base (AB) or the minimum structural section required by the County of San Diego, whichever is more, will be required and shall be considered when a lesser pavement section is indicated by design calculations:

TABLE 14

Design R-value	Design Traffic Index (TI)			
	4.5	5	5.5	6
48	3" AC over 4" AB	3" AC over 4" AB	3" AC over 4" AB	3" AC over 5" AB
Crushed aggregate base (AB) shall meet or exceed the specifications for Class 2 aggregate base materials given in the (Green Book) "Standard Specifications for Public Works Construction," Regional Supplement Amendments, 2006, Sections 400-2.4.				

Final pavement sections will depend on the actual R-value test results performed on finish subgrade soils at the completion of rough grading, design TI and approval of the County of San Diego. All design sections should be confirmed and/or revised as necessary at that time.

Crushed aggregate base materials should be compacted to minimum 95% of the corresponding maximum dry density (ASTM D-1557). Subgrade soils beneath the asphalt paving surfaces should also be compacted to minimum 95% of the corresponding maximum dry density within the upper 12 inches.

2. **PCC Paving:** PCC driveways and parking paving surfaces should be a minimum 5½ inches in thickness, reinforced with #3 reinforcing bars at 18 inches on centers each way placed 2 inches below the top of slab. Subgrade soils beneath the PCC driveways and parking should also be compacted to a minimum 90% of the corresponding maximum dry density within the upper 6 inches.

   In construction practices where the reinforcements are cut at the construction joints, slab panels should be tied together with minimum 18 inches long #3 dowels (dowel baskets) at 18 inches on centers placed mid-height in the slab (9 inches on either side of the joint). In order to enhance performance, tying of the slab panels to the adjacent curbs, where they occur, with #3 dowels at 18 inches on centers may also be considered.

   Provide "tool joint" or "softcut" contraction/control joints spaced 10 feet on center (not to exceed 15 feet maximum) each way. The larger dimension of any panel shall not exceed 125% of the smaller dimension. Tool or cut as soon as the slab will support the weight and can be operated without disturbing the final finish which is normally within 2 hours after final finish at each control joint location or 150 psi to 800 psi. Tool or softcuts should be a minimum of 1-inch in depth but should not exceed 1¼-inches deep maximum. In case of softcut joints, anti-ravel skid plates should be used and replaced with each blade to avoid spalling and raveling. Avoid wheeled equipments across cuts for at least 24 hours.

Joints shall intersect free edges at a 90° angle and shall extend straight for a minimum of 1½ feet from the edge. The minimum angle between any two intersecting joints shall be 80°. Align joints of adjacent panels. Also, align joints in attached curbs with joints in slab panels.

Provide adequate curing using approved methods (curing compound maximum coverage rate = 200 sq. ft./gal.).

3. **General Paving:** Base section and subgrade preparations per structural section design will be required for all surfaces subject to traffic including roadways, travelways, drive lanes, driveway approaches and ribbon (cross) gutters. Driveway approaches within the public right-of-way should have 12 inches subgrade compacted to a minimum of 95% compaction levels, and provided with 95% compacted Class 2 base section per structural section design.

   Base layer under curb and gutters should be compacted to minimum 95%, while subgrade soils under curb and gutters, and base and subgrade under sidewalks should be compacted to minimum 90% compaction levels. Base section may not be required under curb and gutters, and sidewalks in the case of very low expansive subgrade soils (expansion index less than 21) unless otherwise specified. Appropriate recommendations should be given in the final as-graded compaction report.

4. Base and subgrade soils should be tested for proper moisture and specified compaction levels, and approved by the project geotechnical consultant prior to the placement of the base or asphalt/PCC finish surface.

F. General Recommendations

1. The minimum foundation design and steel reinforcement provided herein are based on soil characteristics and are not intended to be in lieu of reinforcement necessary for structural considerations.

2. Adequate staking and grading control is a critical factor in properly completing the recommended remedial and site grading operations. Grading control and staking should be provided by the project grading contractor or surveyor/civil engineer, and is beyond the geotechnical engineering services. Inadequate staking and/or lack of grading control may result in unnecessary additional grading which will increase construction costs.

3. Footings located on or adjacent to the top of slopes should be extended to a sufficient depth to provide the minimum horizontal distance between the bottom edge of the footing and face of slope as specified in this report. This requirement applies to all improvements and structures including fences, posts, pools, spas, etc. Concrete and AC improvements should be provided with a thickened edge to satisfy this requirement.

4. Open or backfilled trenches parallel with a footing shall not be below a projected plane having a downward slope of 1-unit vertical to 2 units horizontal (50%) from a line 9 inches above the bottom edge of the footing, and not closer than 18 inches form the face of such footing.

5. Where pipes cross under-footings, the footings shall be specially designed. Pipe sleeves shall be provided where pipes cross through footings or footing walls, and sleeve clearances shall provide for possible footing settlement, but not less than 1-inch all around the pipe.

6. Foundations where the surface of the ground slopes more than 1 unit vertical in 10 units horizontal (10% slope) shall be level or shall be stepped so that both top and bottom of such foundations are level. Individual steps in continuous footings shall not exceed 18 inches in height and the slope of a series of such steps shall not exceed 1 unit vertical to 2 units horizontal (50%) unless otherwise specified. The steps shall be detailed on the structural drawings. The local effects due to the discontinuity of the steps shall also be considered in the design of foundations as appropriate and applicable.

7. Expansive clayey soils should not be used for backfilling of any retaining structure. All retaining/basement walls should be provided with a 1:1 wedge of granular, compacted backfill measured from the base of the wall footing to the finished surface and a well-constructed back drainage system as shown on the enclosed Plate 9. Planting large trees behind site building/basement retaining walls should be avoided.

8. All underground utility and plumbing trenches should be mechanically compacted to a minimum of 90% of the maximum dry density of the soil unless otherwise specified. Care should be taken not to crush the utilities or pipes during the compaction of the soil. Non-expansive, granular backfill soils should be used. Trench backfill materials and compaction beneath pavements within the public right-of-way shall conform to the requirements of governing agencies.

9. Site drainage over the finished pad surfaces should flow away from structures onto the street in a positive manner. Care should be taken during the construction, improvements and fine grading phases not to disrupt the designed drainage patterns. Roof lines of the buildings should be provided with roof gutters. Roof water should be collected and directed away from the buildings and structures to a suitable location.

10. Final plans should reflect preliminary recommendations given in this report. Final foundations and grading plans may also be reviewed by the project geotechnical consultant for conformance with the requirements of the

geotechnical investigation report outlined herein. More specific recommendations may be necessary and should be given when final grading and architectural/structural drawings are available.

11. All foundation trenches should be inspected to ensure adequate footing embedment and confirm competent bearing soils. Foundation and slab reinforcements should also be inspected and approved by the project geotechnical consultant.

12. The amount of shrinkage and related cracks that occurs in the concrete slab-on-grades, flatworks and driveways depend on many factors the most important of which is the amount of water in the concrete mix. The purpose of the slab reinforcement is to keep normal concrete shrinkage cracks closed tightly. The amount of concrete shrinkage can be minimized by reducing the amount of water in the mix. To keep shrinkage to a minimum the following should be considered:

    * Use the stiffest mix that can be handled and consolidated satisfactorily.

    * Use the largest maximum size of aggregate that is practical. For example, concrete made with ⅜-inch maximum size aggregate usually requires about 40-lbs. more (nearly 5-gal.) water per cubic yard than concrete with 1-inch aggregate.

    * Cure the concrete as long as practical.

    The amount of slab reinforcement provided for conventional slab-on-grade construction considers that good quality concrete materials, proportioning, craftsmanship, and control tests where appropriate and applicable are provided.

13. A preconstruction meeting between representatives of this office, the property owner or planner and grading contractor/builder is recommended in order to discuss grading and construction details associated with the site development.

VIII. LIMITATIONS

The conclusions and recommendations provided herein have been based on available data obtained from the review of pertinent reports and plans, subsurface exploratory excavations as well as our experience with the soils and formational materials located in the general area. The materials encountered on the project site and utilized in our laboratory testing are believed representative of the total area; however, earth materials may vary in characteristics between excavations.

Of necessity we must assume a certain degree of continuity between exploratory excavations and/or natural exposures. It is necessary, therefore, that all observations, conclusions, and recommendations be verified during the grading operation. In the event discrepancies are noted, we should be contacted immediately so that an inspection can be made and additional recommendations issued if required.

The recommendations made in this report are applicable to the site at the time this report was prepared. It is the responsibility of the owner/developer to ensure that these recommendations are carried out in the field.

It is almost impossible to predict with certainty the future performance of a property. The future behavior of the site is also dependent on numerous unpredictable variables, such as earthquakes, rainfall, and on-site drainage patterns.

The firm of VINJE & MIDDLETON ENGINEERING, INC., shall not be held responsible for changes to the physical conditions of the property such as addition of fill soils, added cut slopes, or changing drainage patterns which occur without our inspection or control.

The property owner(s) should be aware that the development of cracks in all concrete surfaces such as floor slabs and exterior stucco are associated with normal concrete shrinkage during the curing process. These features depend chiefly upon the condition of concrete and weather conditions at the time of construction and do not reflect detrimental ground movement. Hairline stucco cracks will often develop at window/door corners, and floor surface cracks up to ⅛-inch wide in 20 feet may develop as a result of normal concrete shrinkage (according to the American Concrete Institute).

This report should be considered valid for a period of one year and is subject to review by our firm following that time. If significant modifications are made to your tentative development plan, especially with respect to the height and location of cut and fill slopes, this report must be presented to us for review and possible revision.

This report is issued with the understanding that the owner or his representative is responsible to ensure that the information and recommendations are provided to the project architect/structural engineer so that they can be incorporated into the plans. Necessary steps shall be taken to ensure that the project general contractor and subcontractors carry out such recommendations during construction.

The project soils engineer should be provided the opportunity for a general review of the project final design plans and specifications in order to ensure that the recommendations provided in this report are properly interpreted and implemented. The project soils engineer should also be provided the opportunity to verify the foundations prior the placing of concrete. If the project soils engineer is not provided the opportunity of making these reviews, he can assume no responsibility for misinterpretation of his recommendations.

Vinje & Middleton Engineering, Inc., warrants that this report has been prepared within the limits prescribed by our client with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Once again, should any questions arise concerning this report, please do not hesitate to contact this office. Reference to our **Job #08-287-P** will help to expedite our response to your inquiries.

We appreciate this opportunity to be of service to you.

VINJE & MIDDLETON ENGINEERING, INC.

Dennis Middleton
CEG #980

S. Mehdi S. Shariat
RCE #46174

Steven J. Melzer
CEG #2362

Distribution: Addressee (5)

DM/SMSS/SJM/vs

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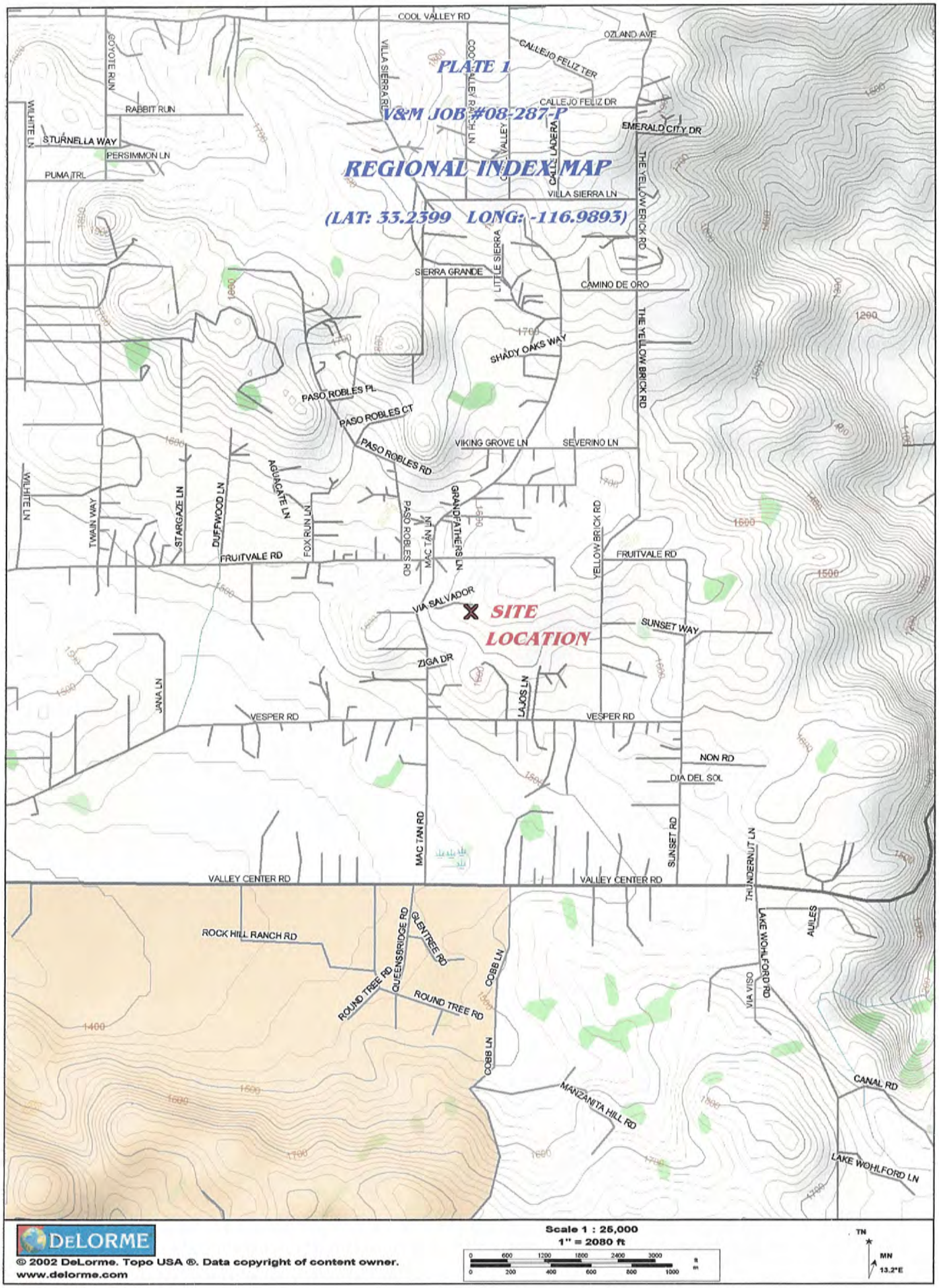
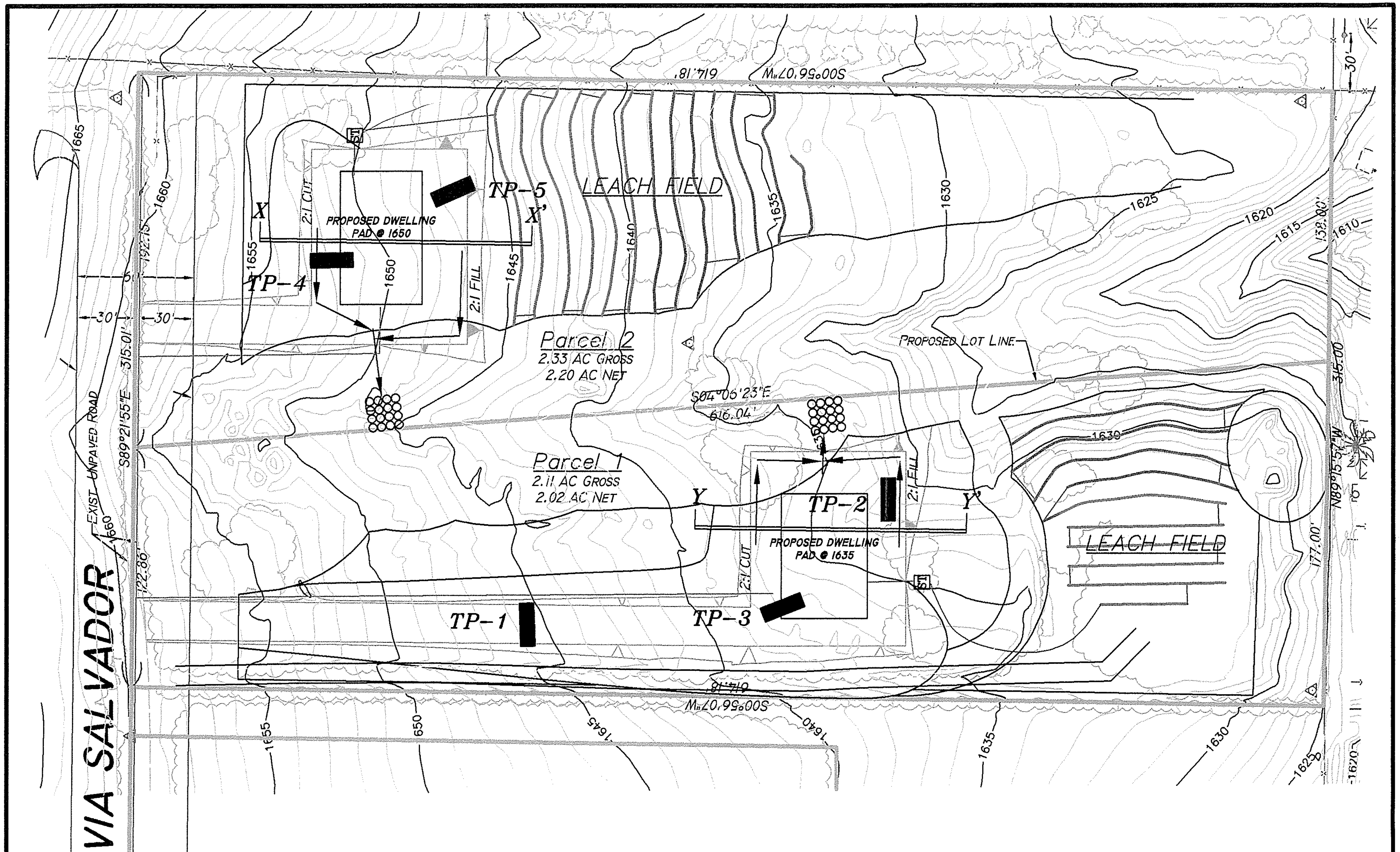


PLATE 1
V&M JOB #08-287-P
REGIONAL INDEX MAP
(LAT: 33.2399 LONG: -116.9893)
SITE LOCATION
COOL VALLEY RD
OZLAND AVE
CALLEJO FELIZ TER
CALLEJO FELIZ DR
EMERALD CITY DR
VILLA SIERRA LN
THE YELLOW BRICK RD
CAMINO DE ORO
SIERRA GRANDE
SHADY OAKS WAY
PASO ROBLES PL
PASO ROBLES CT
PASO ROBLES RD
VIKING GROVE LN
SEVERINO LN
GRANDFATHERS LN
MAC TAN LN
AGUACATE LN
FOX RUN LN
DUFFWOOD LN
STARGAZE LN
TWAIN WAY
WILHITE LN
FRUITVALE RD
YELLOW BRICK RD
VIA SALVADOR
SUNSET WAY
ZIGA DR
LAJOS LN
JANA LN
VESPER RD
NON RD
DIA DEL SOL
MAC TAN RD
SUNSET RD
THUNDERNUT LN
VALLEY CENTER RD
ROCK HILL RANCH RD
QUEENSBRIDGE RD
GLENTREE RD
ROUND TREE RD
COBB LN
LAKE WOHLFORD RD
AUILES
VIA VISO
CANAL RD
MANZANITA HILL RD
LAKE WOHLFORD LN
COYOTE RUN
RABBIT RUN
STURNELLA WAY
PERSIMMON LN
PUMA TRL
VILLA SIERRA RD
COOL VALLEY RANCH LN
CALLE LADERA
Scale 1 : 25,000
1" = 2080 ft
TN
MN
13.2°E

DeLORME



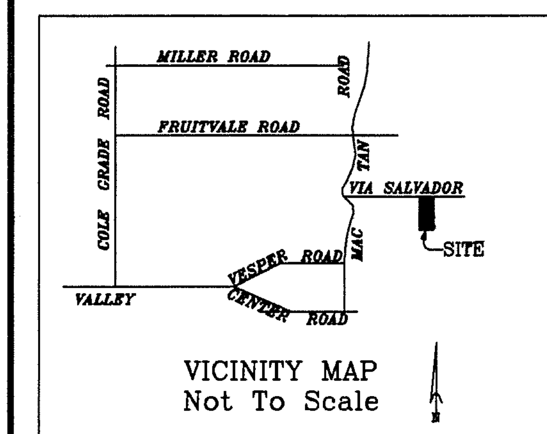
SITE PLAN

EXPLANATION

Approx. Location of Test Pit

Geologic Cross-Section

Scale: 1" = 60'



VICINITY MAP
Not To Scale

PLATE 2
V&M JOB #08-287-P

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

GRAIN SIZES U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

	200		40	10	4		3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY

CLAYS AND PLASTIC SILTS	STRENGTH	BLOWS/FOOT
VERY SOFT	0 - ¼	0 - 2
SOFT	¼ - ½	2 - 4
FIRM	½ - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

1. Blow count, 140 pound hammer falling 30 inches on 2 inch O.D. split spoon sampler (ASTM D-1586)
2. Unconfined compressive strength per SOILTEST pocket penetrometer CL-700

▼ Sand Cone Test ■ Bulk Sample ⊠ $^{2}4_{6}$ = Standard Penetration Test (SPT) (ASTM D-1586) with blow counts per 6 inches

□ Chunk Sample ○ Driven Rings ⊠ $^{2}4_{6}$ = California Sampler with blow counts per 6 inches

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KEY TO EXPLORATORY BORING LOGS
Unified Soil Classification System (ASTM D-2487)

PROJECT NO.		KEY

TEST PIT LOGS

Date Excavated: 6/19/08 **Logged by:** SJM

Equipment: Caterpillar 420 Backhoe **Remarks:** No caving. No groundwater.

TP-1

DEPTH (feet)	SAMPLE TYPE	MATERIAL DESCRIPTION	U.S.C.S.	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
0–3	Bulk sample; Chunk sample	TOPSOIL: Silty fine to medium sand. Brown color. Dry to moist. Loose near surface. Below 1' - Becomes blocky and medium dense to dense. ST-1	SM	6	112.1	84	32
3–3.5		BEDROCK: Gabbroic rock. Coarse grained. Red brown color. Weathered. Gravelly to blocky. Dense. ST-2	SW-GP				
		Bottom of test pit at 3.5 feet.					

Date Excavated: 6/19/08 **Logged by:** SJM

Equipment: Caterpillar 420 Backhoe **Remarks:** No caving. No groundwater.

TP-2

DEPTH (feet)	SAMPLE TYPE	MATERIAL DESCRIPTION	U.S.C.S.	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
0–1		TOPSOIL: Silty fine sand. Brown color. Dry. Loose. ST-1	SM				
1–3.5	Chunk sample	BEDROCK: Gabbroic rock. Fine to coarse grained. Grey color. Weathered. Friable. Somewhat gravelly to blocky. Massive. Dense. ST-2	SW-GP	5	136.5	100+	59
		Bottom of test pit at 3.5 feet.					

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Proposed Two-Lot subdivision
Via Salvador, Valley Center
08-287-P APN 188-321-22

PLATE 3

BULK SAMPLE ■
CHUNK SAMPLE □
SAND CONE ▼
GROUNDWATER ▽

TEST PIT LOGS

Date Excavated: 6/19/08 **Logged by:** SJM

Equipment: Caterpillar 420 Backhoe **Remarks:** No caving. No groundwater.

DEPTH (feet)	SAMPLE TYPE	GRAPHIC LOG	TP-3 MATERIAL DESCRIPTION	U.S.C.S.	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
0–1.3			TOPSOIL: Silty fine sand. Pale brown color. Dry. Loose. ST-1	SM				
1.3–4.5	Bulk (4–4.5)		BEDROCK: Gabbroic rock. Fine to coarse grained. Weathered. Friable. Becomes gravelly below 3'. Massive. Dense. ST-2	SW-GP				
4.5			Bottom of test pit at 4.5 feet.					

Date Excavated: 6/19/08 **Logged by:** SJM

Equipment: Caterpillar 420 Backhoe **Remarks:** No caving. No groundwater.

DEPTH (feet)	SAMPLE TYPE	GRAPHIC LOG	TP-4 MATERIAL DESCRIPTION	U.S.C.S.	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
0–3	Bulk (~1.5); Chunk (~2)		TOPSOIL: Silty fine sand. Brown color. Dry to slightly moist. Firm. ST-1	SM	7	111.1	83	37
3–4	Chunk (~4)		Fine to medium sand with a trace of clay. Olive brown color. Medium dense. Weathered reflection of the underlying bedrock. ST-3	SW-SC	16	106.4	-	74
4–5			BEDROCK: Gabbroic rock. Fine to medium grained. Brown color. Weathered. Friable. Excavates gravelly to blocky below 4.5'. Massive. Dense. ST-2	SW-GP				
5.0			Bottom of test pit at 5.0 feet.					

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PLATE 4

BULK SAMPLE ■
CHUNK SAMPLE □
SAND CONE
GROUNDWATER

TEST PIT LOGS

Date Excavated: 6/19/08 **Logged by:** SJM

Equipment: Caterpillar 420 Backhoe **Remarks:** No caving. No groundwater.

DEPTH (feet)	SAMPLE TYPE	GRAPHIC LOG	TP-5 MATERIAL DESCRIPTION	U.S.C.S.	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	RELATIVE DENSITY (%)	DEGREE OF SATURATION (%)
0–2.5	Chunk sample (at 2)		TOPSOIL: Silty fine sand. Brown to red brown color. Dry. Porous. Loose. ST-1	SM	3	90.4	68	68
2.5–4	Bulk sample (at 4)		BEDROCK: Gabbroic rock. Fine to coarse grained. Olive brown color. Weathered. Excavates somewhat gravelly to blocky. Massive. Dense. ST-2	SW-GP				

Bottom of test pit at 4.0 feet.

Depth scale: 2, 4, 6, 8

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PLATE 5

BULK SAMPLE ■
CHUNK SAMPLE □
SAND CONE ▼
GROUNDWATER ∇

GEOLOGIC CROSS-SECTIONS

SCALE: 1"=20'

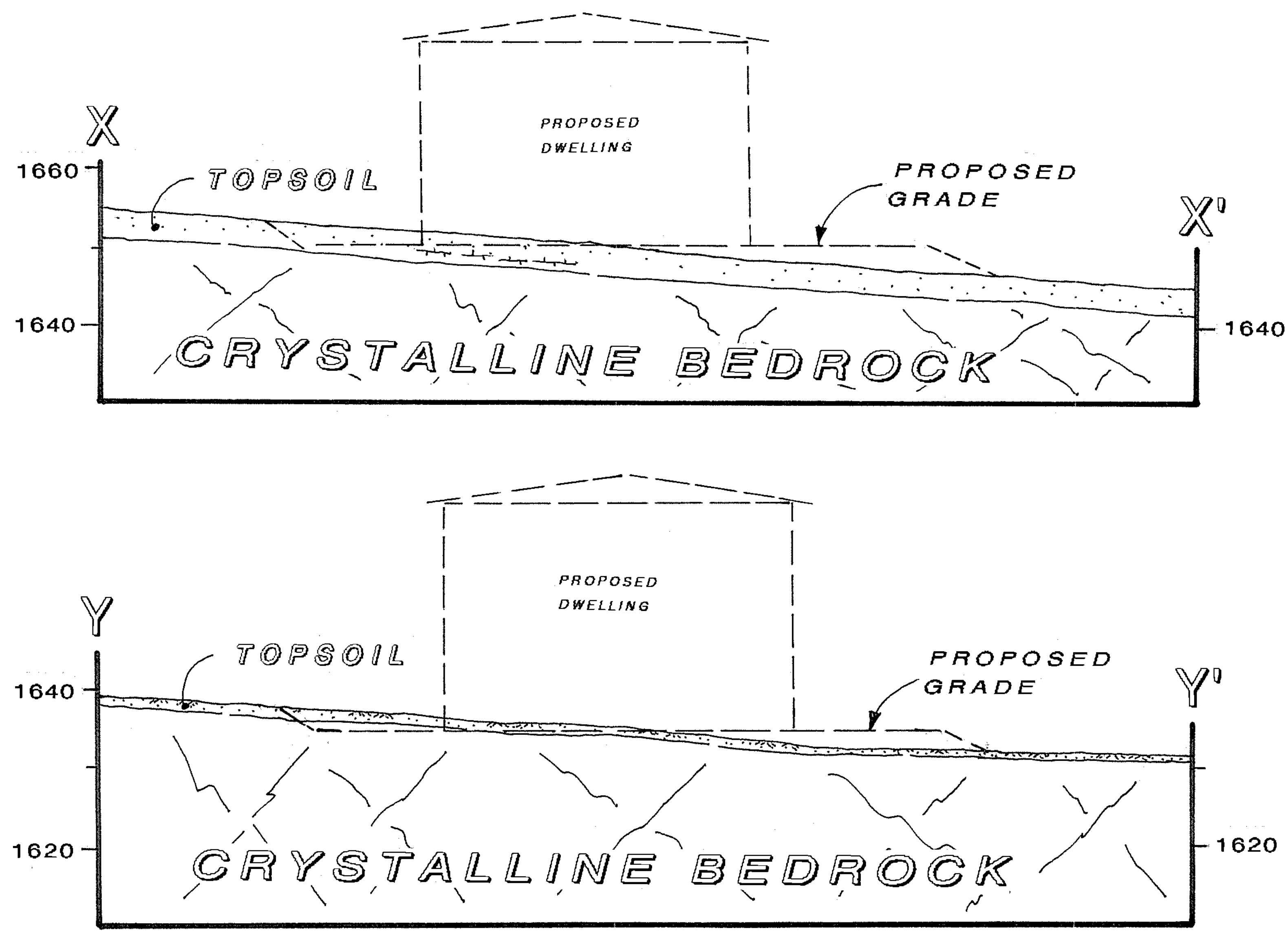
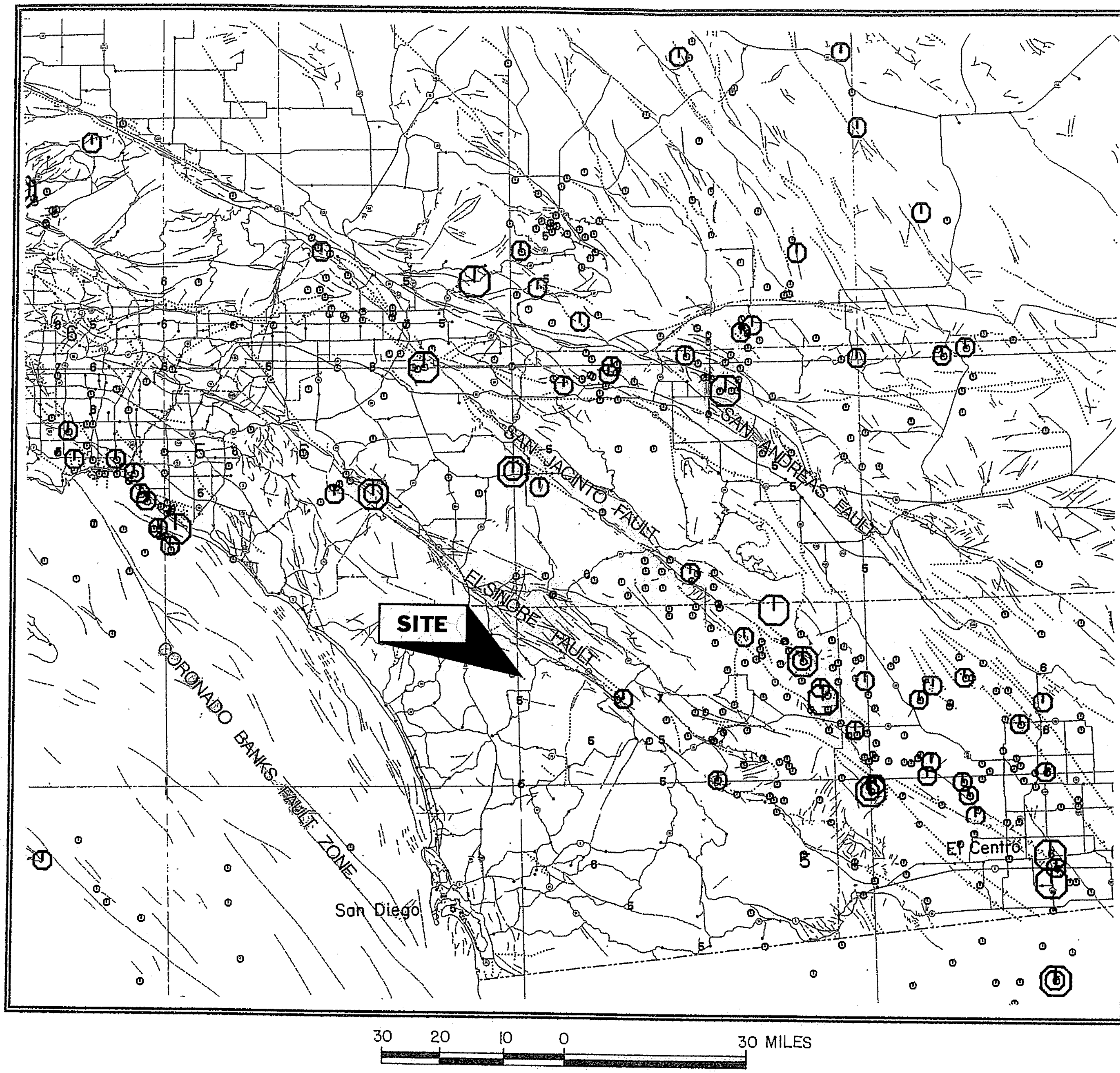


PLATE 6
V&M Job #08-287-P

FAULT - EPICENTER MAP
SAN DIEGO COUNTY REGION



INDICATED EARTHQUAKE EVENTS THROUGH 75 YEAR PERIOD (1900-1974)

Map data is compiled from various sources including California Devision of Mines and Geology, California Institute of Technology and the National Oceanic and Atmospheric Administration. Map is reproduced from California Division of Mines and Geology, "Earthquake Epicenter Map of California; Map Sheet 39." 1978

MAGNITUDE

.... 4.0 TO 4.9
.... 5.0 TO 5.9
.... 6.0 TO 6.9

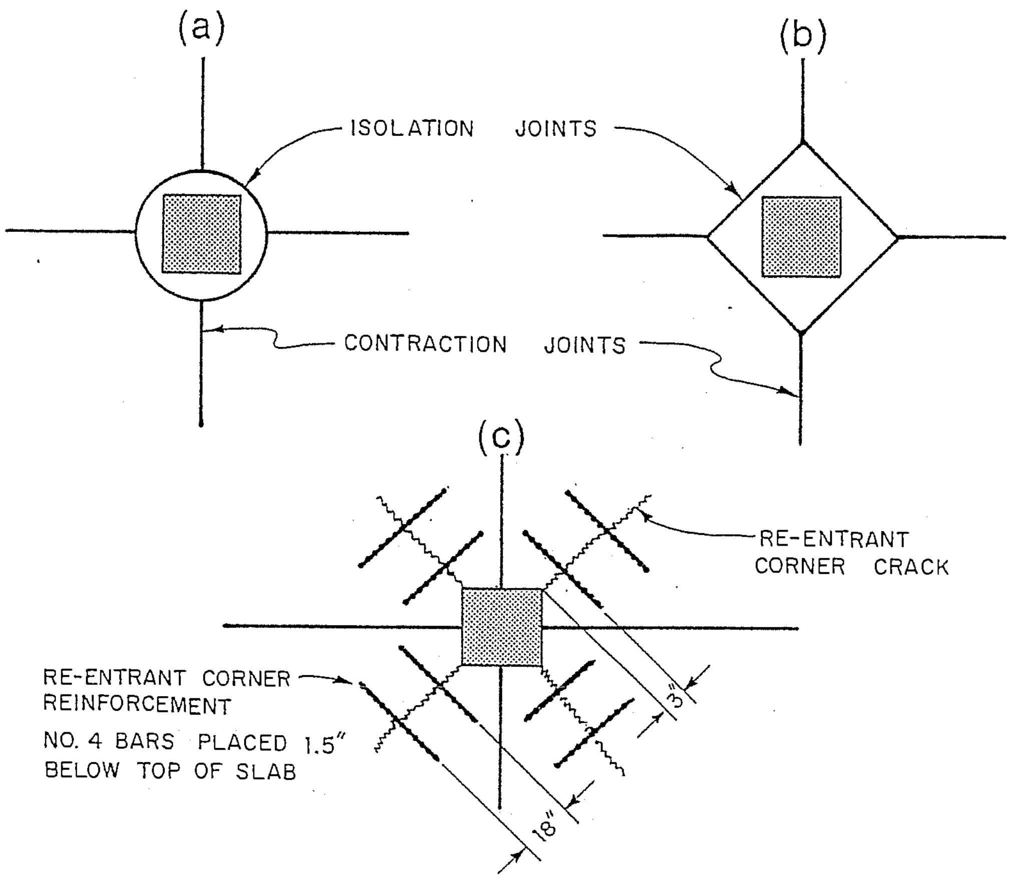
Fault

PROJECT: 08-287-P VIA SALVADOI
VALLEY CENTER

PLATE: 7

ISOLATION JOINTS AND RE-ENTRANT CORNER REINFORCEMENT

Typical - no scale



NOTES:

1. Isolation joints around the columns should be either circular as shown in (a) or diamond shaped as shown in (b). If no isolation joints are used around columns, or if the corners of the isolation joints do not meet the contraction joints, radial cracking as shown in (c)may occur (reference ACI).

2. In order to control cracking at the re-entrant corners (±270° corners), provide reinforcement as shown in (c).

3. Re-entrant corner reinforcement shown herein is provided as a general guideline only and is subject to verification and changes by the project architect and/or structural engineer based upon slab geometry, location, and other engineering and construction factors.

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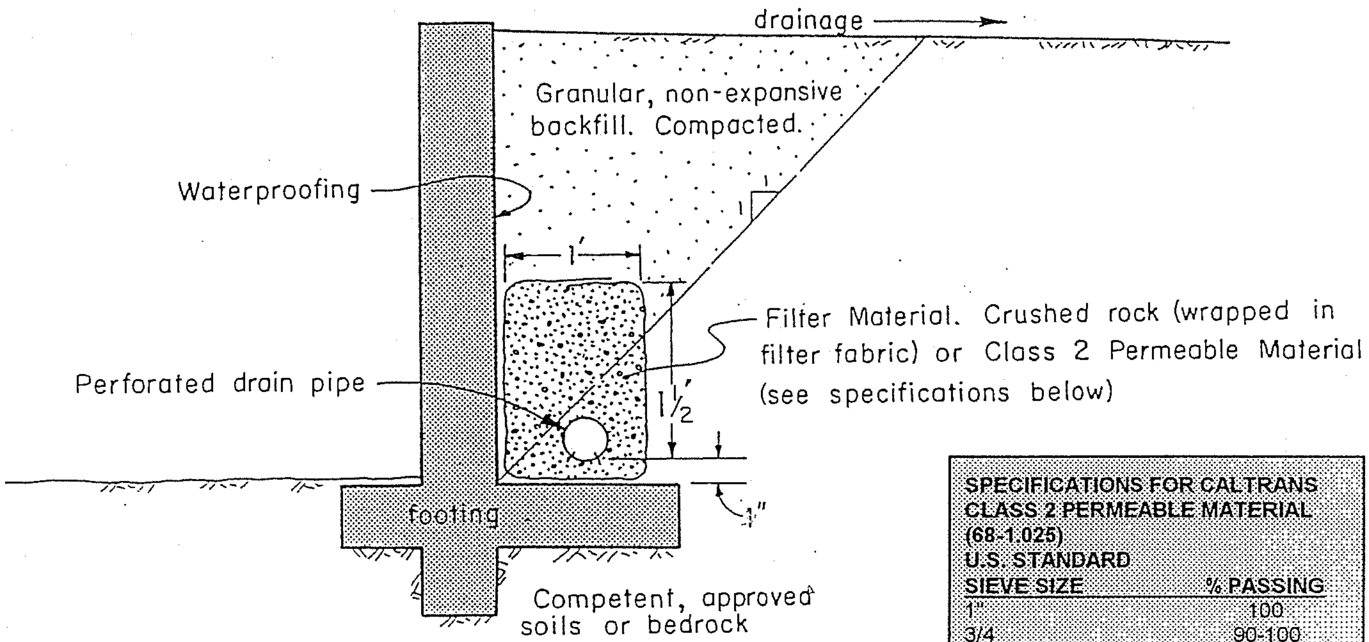
RETAINING WALL DRAIN DETAIL

Typical - no scale



SPECIFICATIONS FOR CALTRANS CLASS 2 PERMEABLE MATERIAL (68-1.025)
U.S. STANDARD

SIEVE SIZE	% PASSING
1"	100
3/4	90-100
3/8	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Sand Equivalent > 75

CONSTRUCTION SPECIFICATIONS:

1. Provide granular, non-expansive backfill soil in 1:1 gradient wedge behind wall. Compact backfill to minimum 90% of laboratory standard.
2. Provide back drainage for wall to prevent build-up of hydrostatic pressures. Use drainage openings along base of wall or back drain system as outlined below.
3. Backdrain should consist of 4" diameter PVC pipe (Schedule 40 or equivalent) with perforations down. Drain to suitable outlet at minimum 1%. Provide ¾" - 1½" crushed gravel filter wrapped in filter fabric (Mirafi 140N or equivalent). Delete filter fabric wrap if Caltrans Class 2 permeable material is used. Compact Class 2 material to minimum 90% of laboratory standard.
4. Seal back of wall with waterproofing in accordance with architect's specifications.
5. Provide positive drainage to disallow ponding of water above wall. Lined drainage ditch to minimum 2% flow away from wall is recommended.

* Use 1½ cubic foot per foot with granular backfill soil and 4 cubic foot per foot if expansive backfill soil is used.

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